

Experimental study on the seismic performance of RC moment resisting frames with precast-prestressed floor units.

B.H.H. Peng, R.C. Fenwick, R.P. Dhakal & D.K. Bull

Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch.



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ABSTRACT: A three dimensional approximately half scale experimental sub-assemblage is currently being tested at the University of Canterbury to investigate the effect of precast-prestressed floor units, which do not span past the internal columns, on the seismic performance of reinforced concrete moment resisting frames. This paper reports the preliminary results from the test, with the focus on elongation within the plastic hinges and strength enhancement in the frames. The preliminary results have shown that elongation between the external and internal plastic hinges varies by more than two fold. With the addition of the prestressed floor units, the strength of the moment resisting frame used in the test was found to be 25% higher than the current code specified value. In other situations, particularly where there are more than 2 bays in a moment resisting frame, greater strength enhancement may be expected. Any under-estimation of beam strength is undesirable as it may result in the development of non-ductile failure modes in a major earthquake.

1 INTRODUCTION

The determination of the flexural over-strength of plastic hinges in beams is a critical issue for the design of moment resisting frames. With the capacity design philosophy used in New Zealand for the design of multi-storey buildings, the structural members are proportioned so that in the event of a major earthquake a ductile beam sway mechanism will form in preference to other less ductile failure modes such as a column sway mechanism. This approach requires the columns to be capable of resisting the maximum actions from the over-strength of the beam plastic hinges. Any over-estimate of the over-strength of the beams increases the structural costs of building while an underestimate could result in a premature collapse of the structure due to the development of a non-ductile column sway mechanism.

Recent research (Fenwick et al. 2006) has found that the determination of reinforced concrete (RC) beam over-strength is not as simple as has been implied in the previous Concrete Structures Standard, NZS3101:1995 (Standards New Zealand. 1995), or in some aspects in the more recent version of this standard, NZS3101:2006 (Standards New Zealand. 2006). The problem arises due to the interaction of floor slabs with beams due to elongation of plastic hinges, especially when either a cast-insitu concrete floor is prestressed or when pretension precast units are used in the construction of the floor. It has been shown that design rules in NZS 3101:1995 were likely to under-estimate the strength of beams by a very considerable margin in two situations (Fenwick et al. 1999). Both of these situations arise where precast prestressed floor units are used and are placed parallel to the beams in a perimeter frame.

The first case was when the precast floor units span past an internal column such as column C2 in Figure 1. In this case, elongation of the plastic hinges adjacent to this column is partially restrained by the floor slab and the prestressed units. This restraint induces tension in the floor slab, which increases the flexural strength on the negative moment side of the column (ie, when flexural tension force is on the top of the beam). Subsequent research at the Universities of Canterbury and Auckland (Matthews 2004; Lindsay 2004; MacPherson 2005; Lau et al. 2007) confirmed that this strength enhancement occurred. Subsequently, a method of assessing this strength increase was developed (Fenwick et al.

2006), and it was included in the current Concrete Structures Standard.

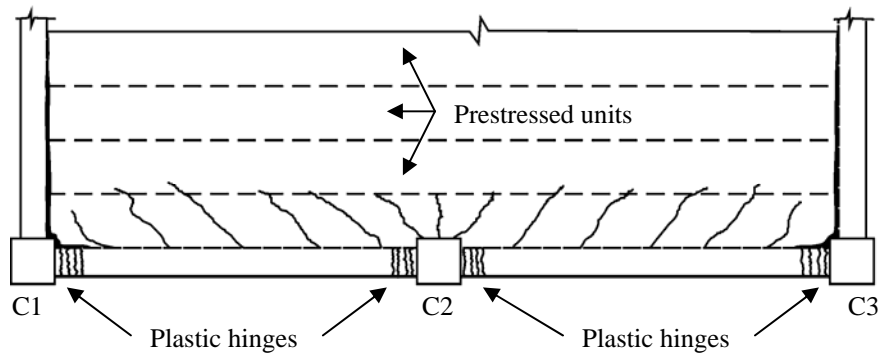


Figure 1. Schematic of prestressed floor units span passed an internal column.

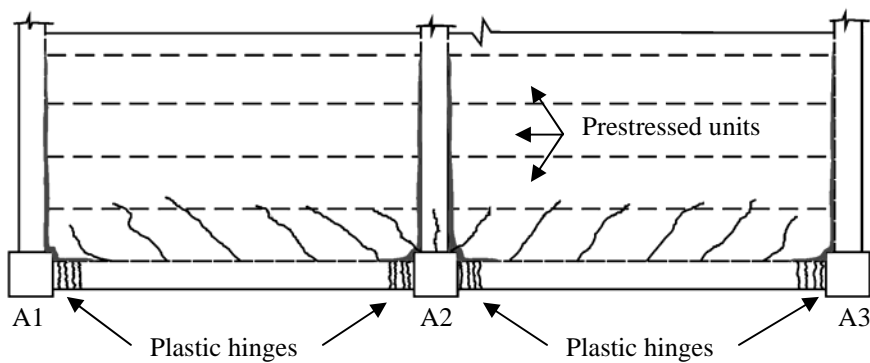


Figure 2. Schematic of prestressed floor units supported on transverse beam framing into an internal column.

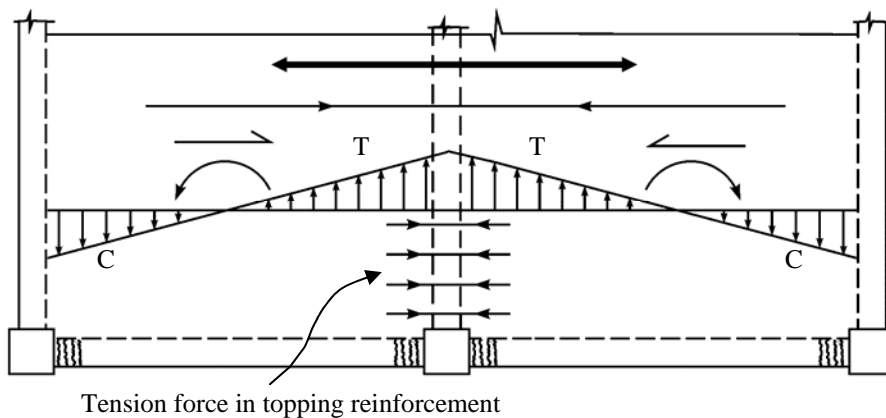


Figure 3. Deep beam actions in floor slabs.

The second case arises when prestressed units are supported on transverse beams framing into each of the columns in a perimeter frame, as illustrated in Figure 2. In this case the prestressed units tie the floor between the transverse beams together. Elongation associated with the plastic hinges adjacent to the internal column pushes the two floor slabs apart so that wide cracks form in the floor at the supports of the precast units. The opening up of these cracks causes the floor slab to act as two deep beams as illustrated in Figure 3. This situation was observed in a sub-assembly test and it was found to result in considerable strength enhancement of the plastic hinges adjacent to the internal columns (Lau et al. 2007). However, the level of strength enhancement from that experiment, or other experimental sub-assemblies with only two bay frames, cannot be used for the many different structural arrangements found in practice. Where there are several bays, the slabs in each bay, acting as a deep

beam, may all contribute to the strength enhancement of the plastic hinges in the beams at the central columns in the perimeter frame. Therefore, an analytical model is needed to determine the strength of the beams associated with different buildings.

This paper describes the preliminary experimental results of a 3D sub-assembly carried out at the University of Canterbury. The test forms part of a project, which has the objective of developing an analytical model that can be used to assess the over-strength of beams in different structural arrangements. The test is still underway at the time of writing. It is intended to use the results from this test to verify the analytical model, which is being developed.

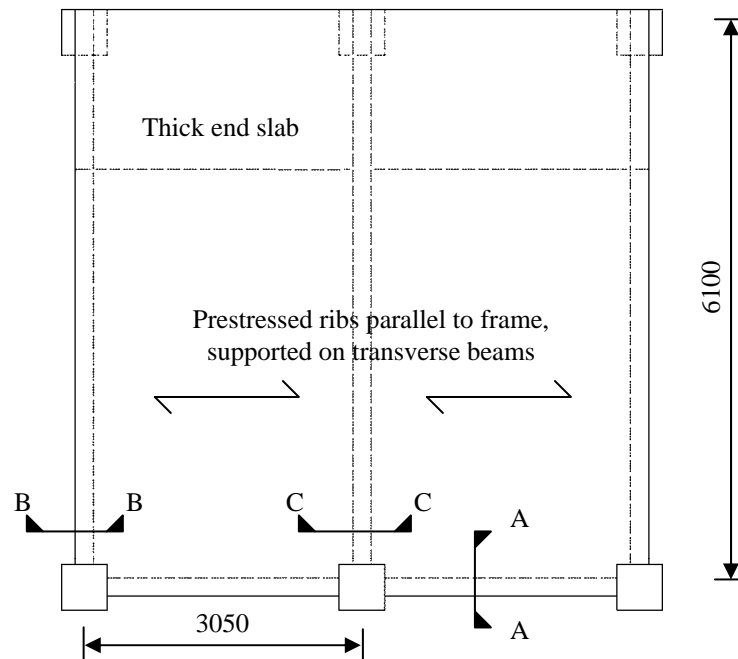
2 EXPERIMENTAL PROGRAMME

2.1 Sub-assembly construction

The experimental sub-assembly simulates a mid-height, corner section of a multi-story RC building. It is a three dimensional, approximately half scaled model, which consists of a two-bay moment resisting frame with transverse beams framing into each column. The key details of the experimental sub-assembly are shown in Figure 4.

The flooring system in the sub-assembly consists of 100mm deep precast prestressed rib units, which span parallel to the perimeter frame and they are spaced at 500mm centres with 45mm concrete topping. The ribs are supported on McDowel bearing strips and 40mm wide ledges on the transverse beams. The reinforcement in the topping concrete consisted of deformed 10mm Grade 300 bars at 210mm centres in both directions, which were lapped with same size starter bars along the perimeter and transverse beams. Timber infills used to support the concrete casting were removed before the test to eliminate the effect of timber on the overall performance of the sub-assembly. The floor was connected to a 175mm thick solid end slab to represent the stiff continuation of a floor diaphragm in the rest of the building.

The sub-assembly was built in four stages. Initially, three bottom columns, three half height transverse beams and full depth longitudinal beams and beam-column joints were constructed. Next, the precast members were erected and the beam-column joints were grouted. The top of the columns and the lap splices connecting the transverse beams to the columns were then cast-in-place. Finally, the prestressed ribs were placed between the transverse beams and the floor slab topping concrete was cast.



(a) Plan

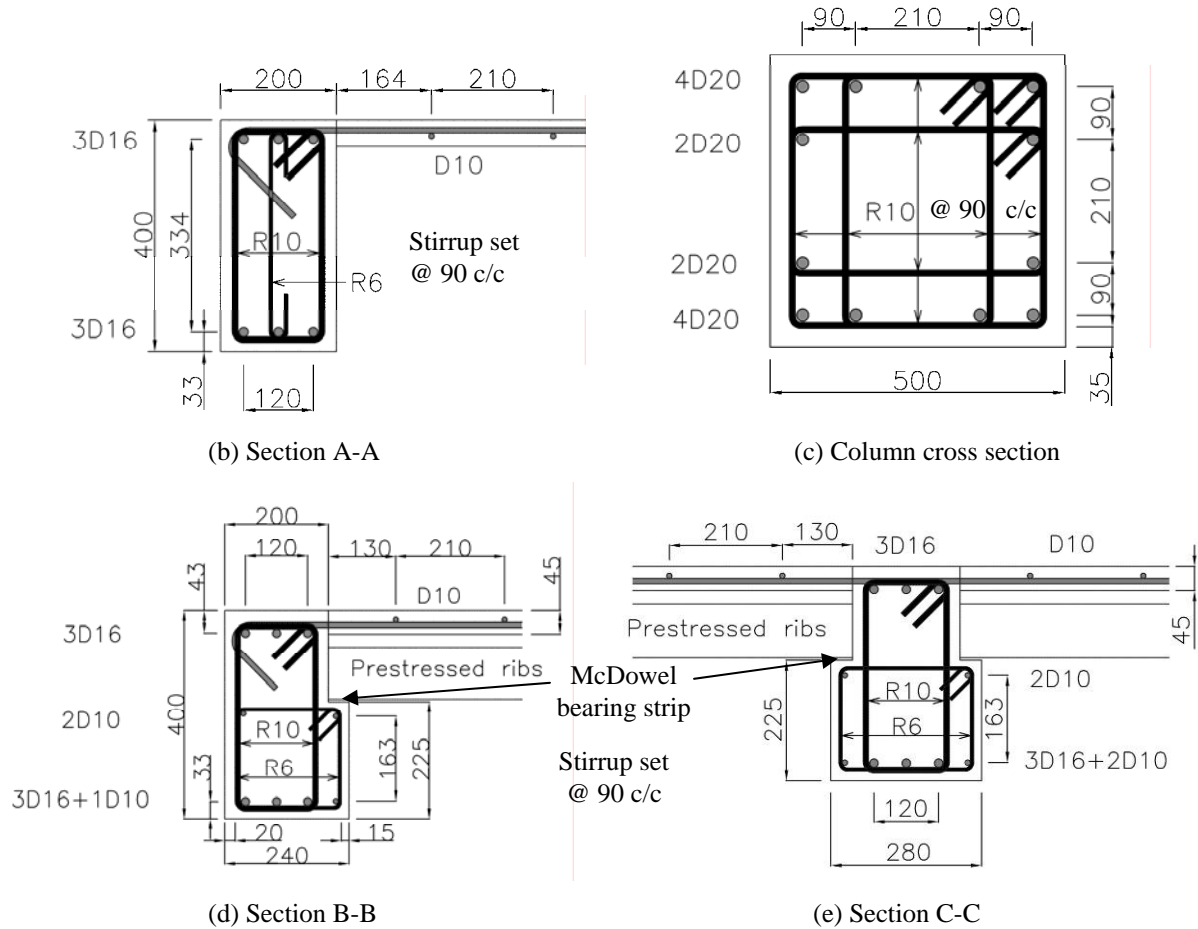


Figure 4. Structural details of the sub-assembly

2.2 Test set-up

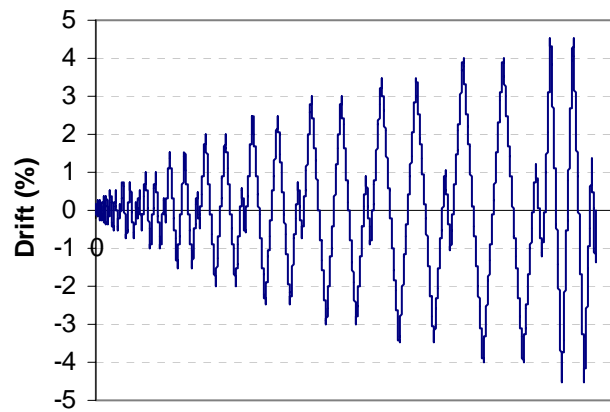
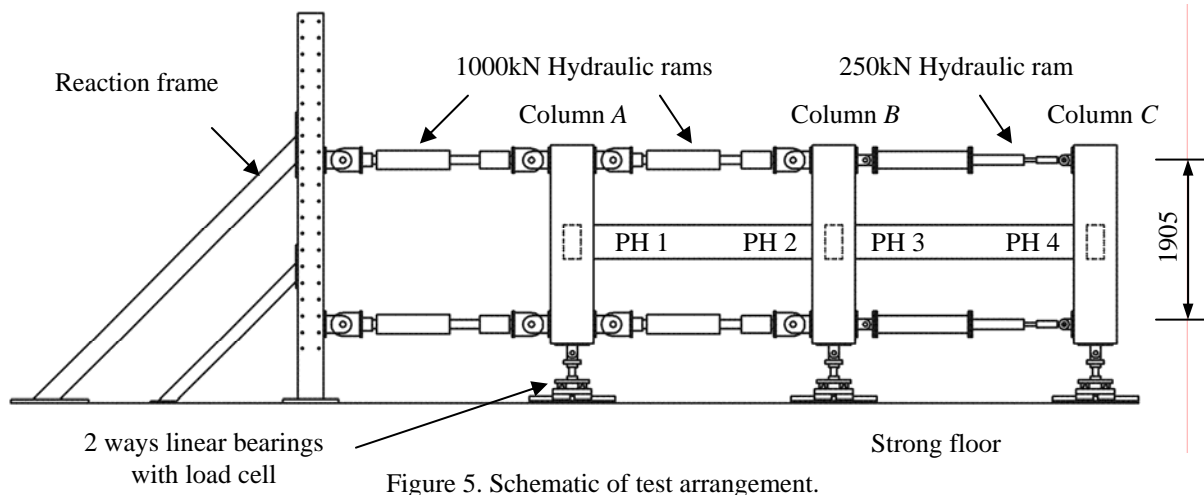
The test set-up, boundary conditions and loading system applied to the sub-assembly were developed to ensure that:

1. Beam elongation is neither restrained or exaggerated;
2. Equal and opposite shear forces are applied to each column;
3. The columns were held parallel to each other during the test;

The loading was displacement controlled. The displacements were applied, in the plane of frame, at the top and bottom of each column, as shown in Figure 5. The columns were supported on two-way linear bearings to allow free elongation of the longitudinal beams. The outer transverse beams were supported on steel columns with one way linear bearing to allow floor movement parallel to frame. The central transverse beam was supported on ball bearings to allow movement in the horizontal plane.

The displacement history applied to the sub-assembly is shown in Figure 6. It followed the displacement history recommended in ACI T1.1-01 (American Concrete Institute. 2001). However, it was felt that applying three cycles at each amplitude, as recommend by ACI T1.1-01, would impose unrealistic and harsh demands on the sub-assembly. Two cycles at each amplitude were applied in the test instead.

Extensive instrumentation was placed on the sub-assembly to measure the lateral and axial forces in the columns and deformations of the beams, columns and floor. The forces were measured using load cells and the deformations were measured using linear potentiometers, rotary potentiometers, inclinometers, sonic displacement transducers, and DEMEC gauges.



3 EXPERIMENTAL RESULTS

3.1 Material properties

The concrete cylinder strengths measured at the start of the test for different casts are summarised in Table 1. The averaged yield stress of the reinforcing bars are summarised in Table 2.

Table 1. Summary of the concrete compressive stress at the start of test.

	Beams, bottom of columns and beam-columns	Top of columns and transverse beam lapped	Floor slabs
Averaged compressive stress (MPa)	31.2	42.4	33

Table 2. Summary of the yield stress of reinforcing bars.

	D10	D16	D20	R6	R10
Averaged yield stress (MPa)	370	321	317	445	391

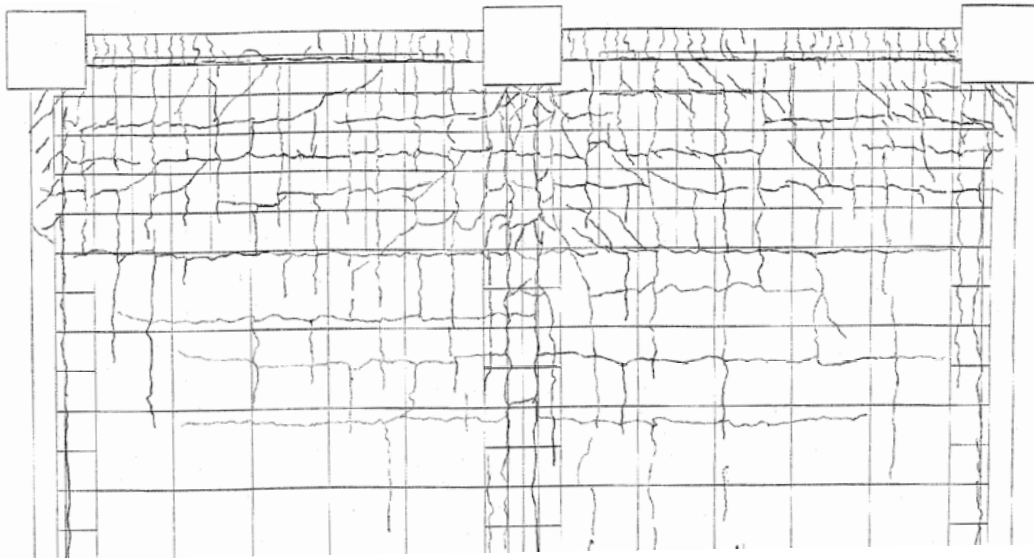
3.2 General observations

A list of important events observed in the experiments are summarised in Table 3.

Table 3. Important events occurring during the experiment

At the end of cycles	Description of key events
0.25%	Minor flexural cracks developed in the beam and floor.
0.35%	Minor flexural cracks developed in the column; diagonal cracks emerged in the beam; minor cracks developed at the connections between the prestressed ribs and the transverse beams.
0.5%	Diagonal cracks developed in the topping concrete.
0.75%	First sign of yielding in beam between columns <i>A</i> and <i>B</i> in Figure 5; minor diagonal cracks developed in the central beam-column joint.
1.0%	Torsional cracks developed in the transverse beams; minor spalling occurred between the beam to column interface.
1.5%	Minor spalling occurred in the plastic hinges; cracks opened up in the floor next to the column face by 5mm; vertical differential movement between floor and transverse beam was apparent.
2.0%	External columns, <i>A</i> and <i>C</i> , were observed to twist.
2.5%	The first floor starter bars parallel to frame partially lost the concrete cover; moderate spalling in the plastic hinges.

The crack pattern in the floor slab at the end of 1.5% drift cycles is shown in Figure 7. The diagonal cracks inclined towards the central column indicate the floor slab partially restraint elongation in the plastic hinges. The longitudinal cracks parallel to frame, near the end slab, shows the deep beam actions of the floor.

**Figure 7. Floor slab crack patterns at the end of 1.5% drift**

The total lateral force versus drift is plotted in Figure 8. It can be seen that flexural strength of the frame developed roughly at 0.75% drift and that the increase in lateral force between this drift and the drift at 2.5% is in the order of 30 percent.

Elongation in the external and internal plastic hinges is plotted in Figure 9a and b. It can be seen that the growth in the external plastic hinges is more than twice the growth in the internal plastic hinges. This is expected as the floor slabs provide more restraint to the growth in the internal plastic hinges.

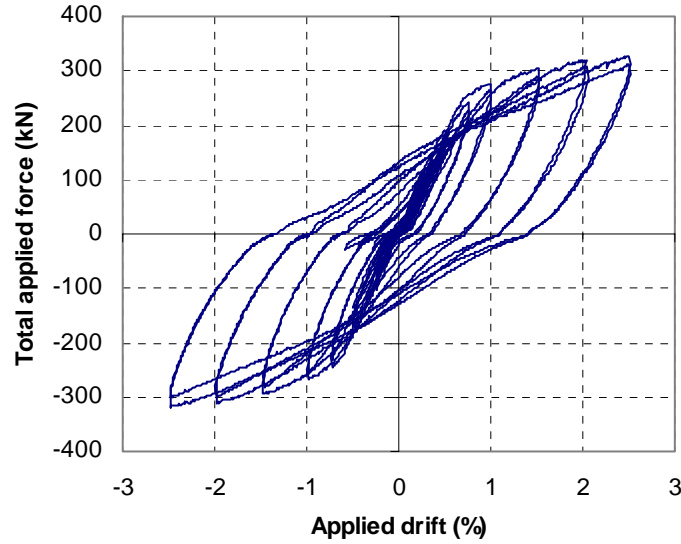


Figure 8. Force displacement relationship of the frame.

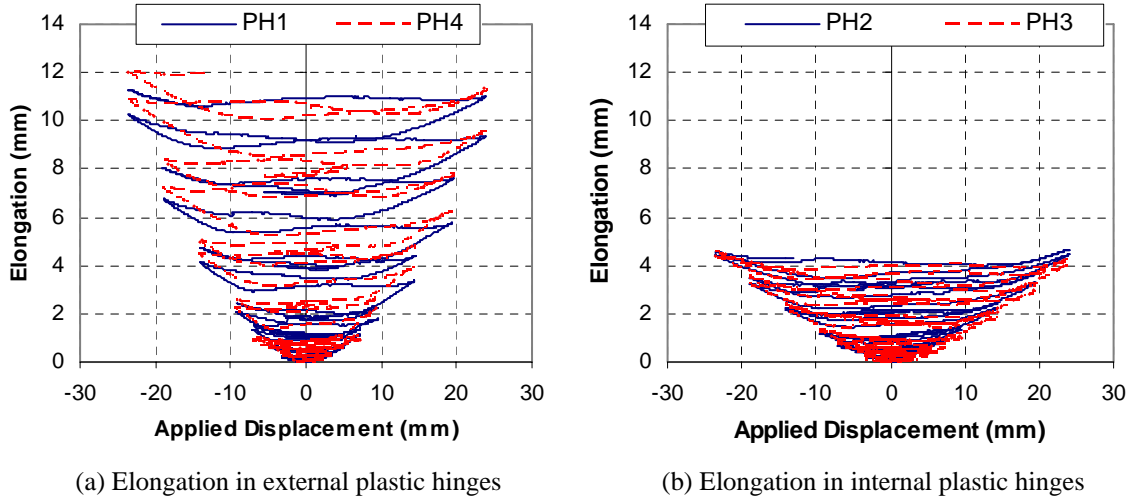


Figure 9. Elongation in the beam plastic hinges

3.3 Contribution of floor slab to the theoretical and over-strength of beams

The theoretical flexural strength of the beam with slab was determined using the appropriate effective compression and tension flange widths specified by NZS3101:2006 in clauses 9.3.1.2 and 9.3.1.4 respectively. The positive and negative theoretical flexural strength calculated with measured material properties were 71.7 and 84.2kNm respectively. The corresponding shear forces in columns *A*, *B* and *C* (see Figure 5) when a positive drift was applied are given in Table 4.

The over-strength of the beams was calculated using the effective flange width defined in clause 9.4.1.6.2. The over-strength value of 1.25 for Grade 300 steel specified in the code assumes a 15% increase in yield stress based on the statistical upper 95 percentile value plus a strain-hardening influence of 10%. In the experiment, the actual material yield stress was measured and hence an over-strength value of 1.1 was used for calculating the over-strength of the beams. The negative over-strength moment for PH2 and PH4 in Figure 5 are 141.1 and 124.5kNm respectively and the positive over-strength moment is 78.3kNm when a positive drift was applied. The corresponding column shear force is given in Table 4.

Table 4. Comparisons of theoretical strength and strengths measured in the test.

	Column average shear force (kN)			Total average shear force(kN)
	Column A	Column B	Column C	
Theoretical strength	45.4	97.9	52.5	195.8
Over-strength	52.1	136.9	76.1	265.1
1 st cycle at +0.5 %	56.3	93.2	43.2	192.7
1 st cycle at +0.75 %	62.4	119.1	59.7	241.2
1 st cycle at +1.0 %	65.5	139.1	72.9	277.5
1st cycle at +1.5 %	71.5	155.3	78.9	305.7
1st cycle at +2.0 %	74.7	163.3	82.9	320.9
1st cycle at +2.5 %	76.3	167.9	85.5	329.7

From these comparison, it can be seen that both the theoretical and over-strength of the beams are underestimated using the design criteria in the current standard. The experimental theoretical strength, developed at a drift of 0.75%, and the over-strength are 23% and 25% higher than the value specified by the code. The amount of over-strength enhancement arising from each column was 24kN, 31kN and 9.4kN respectively. It is not clear at this stage why there was appreciable increase in strength in the positive moment at the external columns.

4 DISCUSSIONS AND CONCLUSIONS

The preliminary experimental result shows that a slab containing prestressed floor units can increase the strength of RC moment resisting frames by acting as deep beams to restrain elongation in plastic hinges in perimeter frames. Current provisions in NZS3101:2006 appear to underestimate both the theoretical and over-strength of the beams. In multi-bay frames, greater strength increases due to this action could be anticipated. In this situation each bay may act as a deep beam to increase the restraint on the plastic regions in the central portion of a frame.

The whole basis of capacity design depends on being able to assess the strength of primary plastic regions. The test results show the importance of the interaction of floor slabs and beams in assessing member over-strengths and they indicate further research is required to identify the strength increase.

An analytical model is required to determine the level of strength enhancement associated with the interaction between prestressed floor units and frames for different structural arrangements found in practice.

Acknowledgements

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